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LATERAL LOAD ON SINGLE PILE: A COMPARATIVE STUDY BETWEEN EGYPTIAN CODE, CANADIAN CODE, AASHTO, INDIAN STANDARDS AND DIFFERENT CODES AND METHODS

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ABSTRACT

The lateral loading pile problem is frequently solved by assuming that the pile is an elastic element and that the soil may be represented by a sequence of nonlinear horizontal springs. The P-y curves illustrate the soil springs' nonlinear behaviour. There are many ways, in literature and adopted in codes to calculate the displacement, ultimate, and allowable lateral load resulting from applying horizontal forces on a vertical pile. The aim of this study is to determine which codes and equations give the most accurate result in ultimate and allowable lateral load for available measurements and case studies. A study was conducted on 40 field and laboratory experiments under different soil conditions and different locations. The Egyptian, Canadian, British standard, AASHTO, DIN 4014 and Indian standards, as well as the Jean-Louis-Briaud method were used to determine the ultimate, and allowable lateral load values and compare them with field and laboratory measurements. The Canadian, British standard, and AASHTO, use broom's method to calculate ultimate, and allowable lateral load **[1] [2] [3] .**

German code uses an approximate method to find the maximum lateral displacement of 2cm or equal to 0.03D **[4] .**

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1. INTRODUCTION

In engineering practice, the lateral capacity of piles is frequently a significant component. As a result, design engineers must take it into consideration. When employed beneath towering chimneys, high-rise buildings, highway bridges, and coastal and offshore projects, piles are usually subjected to significant lateral stresses in addition to the enforced vertical loads.

Earthquakes, wind, ship collision, wave action, landslip pressures, and traffic can all cause lateral loads.

Considerable theoretical and experimental efforts have been done to predict and analyse the behaviour of piles under lateral loads. One of the first attempts to explain pile lateral behaviour was made by Matlock and Reese (1960) **[5].** They devised a method for determining soil reaction, pile deflection, and bending moment along the pile. (Broms 1964a, b) **[6]** described a method for evaluating the lateral behaviour of short, intermediate, and long piles under free-head and fixed-head situations.

The lateral pile resistance is determined by the stiffness and strength of the pile material and the soil in the top zone surrounding the pile head.

40 field and laboratory experiments data collected from many sources (**[7] [8] [9] [10] [11] [12] [13])** with different soil conditions, such as sand fill over clay, silty sand, Medium sand, soft clay, Medium clay, sand and clay soil, were used in this study to find the horizontal displacement resulting from horizontal lateral load **.**

The equations and coefficients of the Egyptian, Canadian, British standard, AASHTO, DIN 4014 and Indian codes were used, as well as the Jean-Louis-Briaud method were used to compare whichever gives the closest and most accurate result to field and laboratory experiments.

2. EGYPTIAN CODE ECP 202 (2005) [14]

Two different methods for designing piles according to the Egyptian code are presented there after.

2.1 FIRST METHOD USING A HORIZONTAL SOIL REACTION COEFFICIENT [14]

In this method the piles are designed as follows:

First, the value of the horizontal soil reaction coefficient is calculated.

In the case of soils with a k_h fixed value with depth, such as over consolidated clays, the value ranges between 35-70 times the undrained shear strength value.

In case of soils in which the coefficient of horizontal soil reaction increases with the depth below ground surface of the earth, the relationship given in the Egyptian code is use as shown in **equation (1):**

$$K_h = \frac{n.z}{d} \qquad (1)$$

Where

Kh: Soil reaction coefficient.

n: Coefficient according to soil type.

z: Depth

d:Diameter of pile.

The relative stiffness of the pile is calculated in terms of the so-called elastic length from one of the following **equations (2),(3):**

$$\mathbf{Io} = \sqrt[4]{\frac{4EI}{kh \cdot d}} \qquad (2)$$

$$\mathbf{t} = \sqrt[5]{\frac{EI}{n}} \qquad (3)$$

where
Io: The relative hardness of the pile when kh constant.
Kh: Soil reaction coefficient.
E: young's modulus of pile.
I:moment of inertia of pile.
d: Diameter of pile.
t: The relative hardness of the pile when kh variable.
n: Coefficient according to soil type.

(Hetenyi,1946) **[14]** equations are used to calculate the displacements and stresses on the soil and the expected bending moments of the pile.
The pile is considered very stiff if one of the two conditions is met:

$$\frac{L}{I_0} \leq 1 \quad \text{OR} \quad \frac{L}{t} < 2$$

The pile is considered highly flexible if one of the two conditions is met:

$$\frac{L}{I_0} \geq 3 \quad \text{OR} \quad \frac{L}{t} > 4$$

Where :
L: length of pile.
Io: The relative hardness of the pile when kh constant.
t: The relative hardness of the pile when k_h variable.

In the case of highly flexible piles, the maximum displacements and the expected bending moments of the pile can be calculated from some tables from the Egyptian code.
In the case of highly stiff piles, the maximum displacements and bending moments can be calculated using (Barber,E.S, 1953) **[14]** equations.

In the case of piles of medium stiffness, refer to the following reference: (Duncan,J.M.& Ooi,P.S.K,1994) **[14].**

2.2 SECOND METHOD CONSIDERING THE SOIL AS A FLEXIBLE MEDIUM. [14]

The pile is designed for horizontal load in this method using the (Poulos&Hull 1989) **[14]** method, in which the flexible and rigid pile is designed according to the loading state and the state of the pile head. The pile is classified as flexible or stiff according to the critical length and is calculated for the homogeneous and nonhomogeneous soil with elastic modulus along the depth of the pile. From the following **equations (4) ,(5) ,(6) ,(7) ,(8)**:

$$\mathbf{Lc} = 4.44\sqrt[4]{\frac{EI}{E_s}} \qquad (4)$$

$$\mathbf{Lc} = 3.3\sqrt[5]{\frac{EI}{n_h}} \qquad (5)$$

$$\rho = \frac{H\,I_1}{E_e\,L_e} + \frac{I_2 M_O}{E_e L_e^2} \qquad (6)$$

$$\theta = \frac{H\,I_2}{E_e L_e^2} + \frac{I_3 M_O}{E_e L_e^3} \qquad (7)$$

$$I = A + B\log(L_e\,/\,d) \qquad (8)$$

Where:

Lc: Critical length.
E: young's modulus of pile.
I: moment of inertia of pile.
E_s:Modulus of elasticity of soil.
n_h: The rate of increase of the modulus of elasticity with depth
ρ : displacement for pile.
θ : rotation for pile.
H : Horizontal load at the pile surface
E_e: The modulus of elasticity of soil through a depth equal to the effective length .
L_e: Effective length of pile.
M_O: moment on pile.
I_1, I_2, I_3, I_4, I_5: Coefficients depend on the effective length and diameter.

Table (1) gives the Coefficients for calculating displacement and moments for laterally loaded piles constructed in homogeneous or nonhomogeneous soil.

Table (1) Coefficients for Behaviour of Pile (After Egyptian code 203 (2020))

Type of pile	RELATION OF EMBEDDED LENGTH WITH STIFFNESS FACTOR	
	linearly increasing	constant
SHORT (RIGID) PILE	$L \leq 2T$	$L \leq 2R$
LONG (ELASTIC) PILE	$L \geq 4T$	$L \geq 3.5R$

3. Indian standard IS 2911 (2010) approach [15]

IS 2911 was used to investigate the behaviour of laterally loaded piles. Because of the intricacy of many issues, the IS technique always produces an approximate answer. The first stage was to figure out if the pile was a small, rigid unit or an indefinitely long, flexible part. This was accomplished by determining the stiffness factor, T, for a given pile and soil combination. Having The stiffness factor was computed, and the behaviour conditions were determined as follows:

The terms "short stiff pile" and "long elastic pile" are both used to describe piles that are either rigid or elastic as shown in **Table (2)**.

L of the pile's embedded length the distance the depth from the ground surface to the point of virtual fixity used to estimate elastic properties in the classical elastic analysis Bending moment and lateral deflection.

The lateral soil resistance of granular soils and typically consolidated clay with changing soil modulus was determined using the modulus of subgrade response, as illustrated in **Tables (3) and (4)**.

The stiffness factor T for granular soils and the stiffness factor R for cohesive soils were calculated using the modulus of subgrade response, as shown in **equations (9) and (10).**

case	I	HOMOGENOUS SOIL		MULTI SOIL	
		A	B	A	B
ELASTIC PILE	I1	1.646	3.395	13.1	11.09
	I2	5.52	9.082	34.63	18.03
	I3	64.98	37.95	159.1	37.14
	I4	1.326	1.641	5.659	4.136
	I5	0.098	0.042	0.228	0.04
RIGID PILE	I1	0.098	2.196	3.181	9.701
	I2	0.701	3.225	2.409	12.71
	I3	1.076	6.297	1.844	18.65
	I4	0.539	0.545	0.773	1.081
	I5	0.547	-0.014	0.764	-0.34

$$T = \sqrt[5]{\frac{EI}{\eta h}} \qquad (9)$$

$$R = \sqrt[4]{\frac{EI}{KB}} \qquad (10)$$

Table (2) The terms "short stiff pile" and "long elastic pile" (After Indian standard IS 2911 (2010))

Table (3) Modulus of Subgrade Reaction for Granular Soils, ηh, in

kN/m^3 (After Indian standard IS 2911 (2010))

SI.NO	Soil type	N (blows/30cm)	Range of ηh in KN/m3*103	
1	**Very loose sand**	0-4	<0.4	<0.2
2	**Loose sand**	4-10	0.4-2.5	0.2-1.4
3	**Medium sand**	10-35	2.5-7.5	1.4-5
4	**Dense sand**	>35	7.5-200	5-12

Table (4) Modulus of Subgrade Reaction for clay Soils, K, in kN/m^3 (After Indian standard IS 2911 (2010))

finally, to calculate deflection the **equations (11) and (12)** are used.

Y= $\frac{H(e+z_f)^3}{3EI}$ **free head pile** **(11)**

Y= $\frac{H(e+z_f)^3}{12EI}$ **fixed head pile** **(12)**

where

H = lateral load, in kN;

y = deflection of pile head, in mm;

E = Young's modulus of pile material, in kN/m^2;

I = moment of inertia of the pile cross-section,in m^4;

z_f = depth to point of fixity, in m; and

e = cantilever length above ground/bed to the point of load application, in m.

And, to calculate moment the **equations (13) and (14)** are used.

Mf = H (e + Zf) free head pile **(13)**

Mf = $\frac{H(e+z_f)}{2}$ **fixed head pile** **(14)**

The equivalent cantilever's fixed end moment, MF, is greater than the pile's real maximum moment M. The real maximum duration may be calculated by multiplying the fixed end with by a

SI. NO.	Soil consistency	Unconfined compression strength qu (kn/m2)	Range of K in KN/m3*103
1	**Soft**	25-50	4.5-9
2	**Medium stiff**	50-100	9-18
3	**Stiff**	100-200	18-36
4	**Very stiff**	200-400	36-72
5	**Hard**	>400	>72

decrease in the comparable cantilever's moment m, as seen in **Figure (1).**

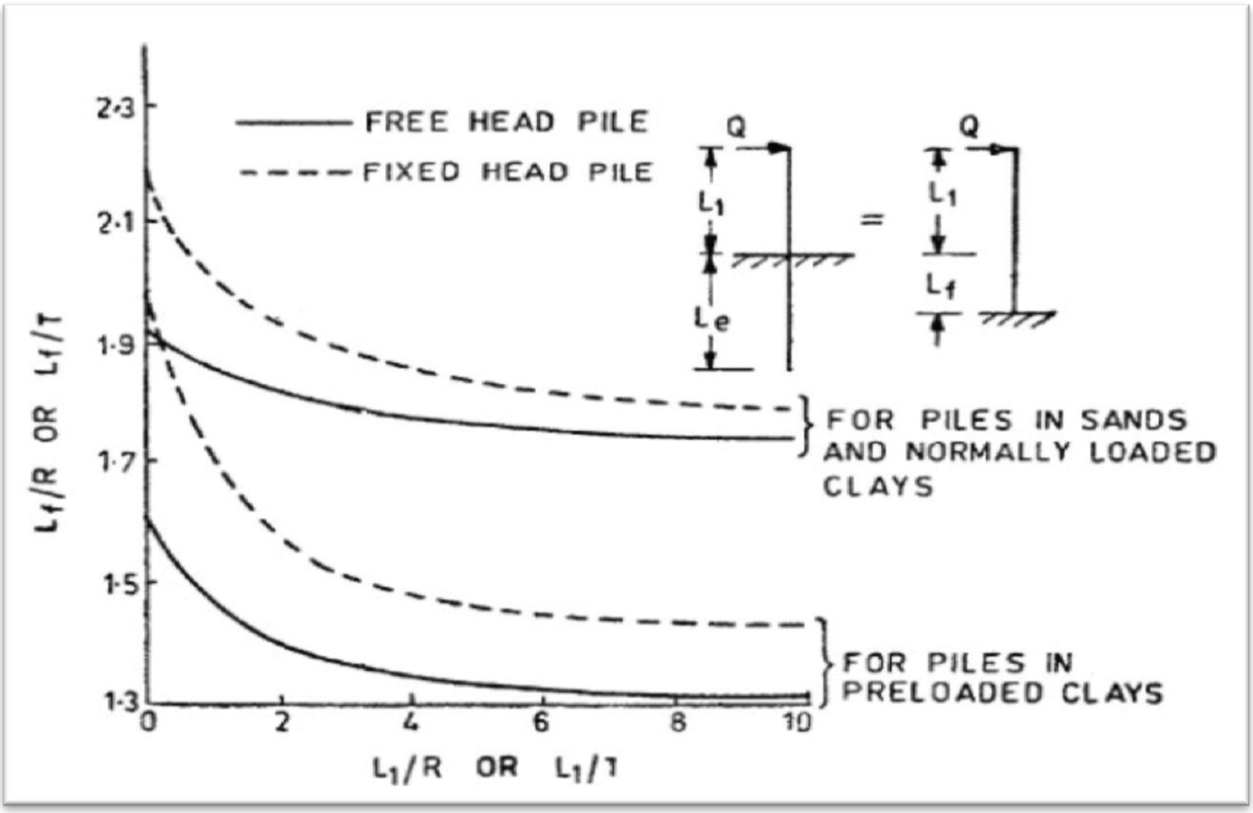

Fig. (1): Depth Of Fixity (After Indian standard IS 2911 (2010))

4. BRITISH STANDARD (2002), AASHTO LRFD BRIDGE (2005), AND CANADIAN CODE [1][2][3]

By deflecting until the required response in the surrounding soil is mobilized, vertical piles withstand lateral loads or moments. The stiffness of the pile and the strength of the soil have a major role in the foundation's behaviour under such loading situations. The horizontal load capacity of vertical piles can be limited in three ways: the soil's capacity can be exceeded, resulting in large horizontal pile movements and foundation failure; bending moments and/or shear can cause excessive bending or shear stresses in pile material, resulting in pile structural failure; or the pile heads' deflections can be too large to be compatible with the superstructure. Design must take into account all three types of failure. These design approaches may still be improved, and the optimum way is frequently still based on well-planned and conducted lateral test loads. British Standard (2002), AASHTO LRFD Bridge (2005), and Canadian code all of them use Broom`S method in calculate deflection and horizontal load.

4.1.BROMS`S METHOD[1][2][3]

Various static evaluations of lateral load capacity, including those of Brinch-Hansen, have been recorded (1961). Broms (1964) has offered solutions for homogeneous clay and sand strata in graphical form **Figures (2),(3),(4)and (5).** In each example, two forms of pile failure are investigated: ‘short' pile failure, in which the soil near to the pile's lateral capacity is entirely mobilised, and 'long' pile failure, in which the pile's bending resistance is fully mobilised.

The solutions are based on a set of simplifying assumptions about the size and distribution of lateral soil pressures along the pile. A pile with a diameter of d and an embedded length of L; lateral load capacity Pu; pile yield moment, Myield; clay cohesiveness, cu; coefficient of passive sand resistance, Kp; height of lateral load above groundline, e; and soil unit weight, y are all discussed.

Poulos (1985) has added lateral load capacity to Broms' solutions for piles in layered clay soils.

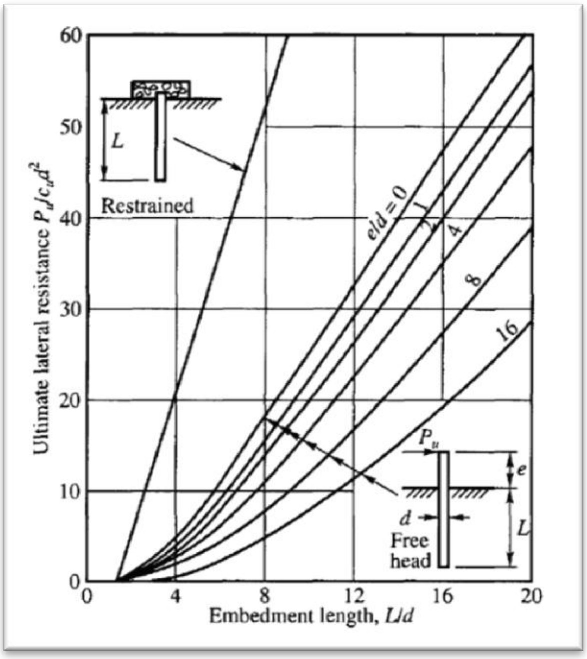

Fig. (2): for cohesionless soil when pile is short.(After (Broms 1964))

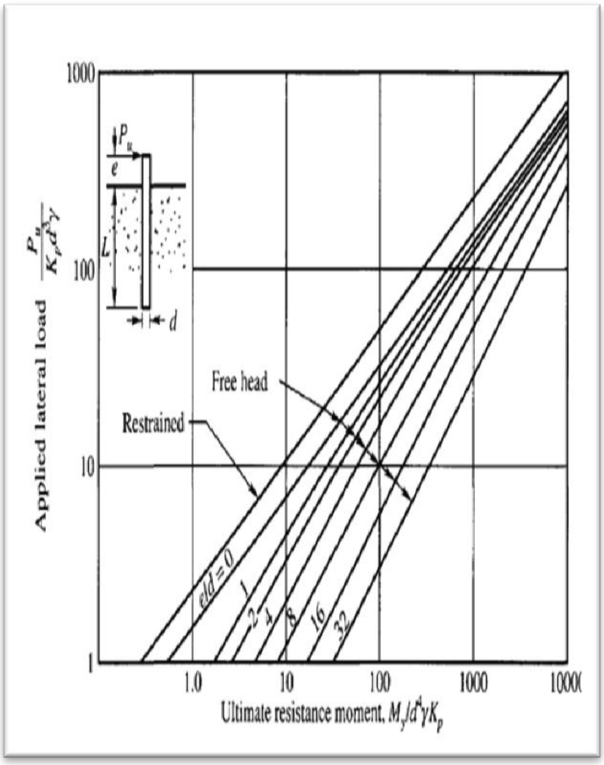

Fig. (3) for cohesionless soil when pile is long. (After (Broms 1964))

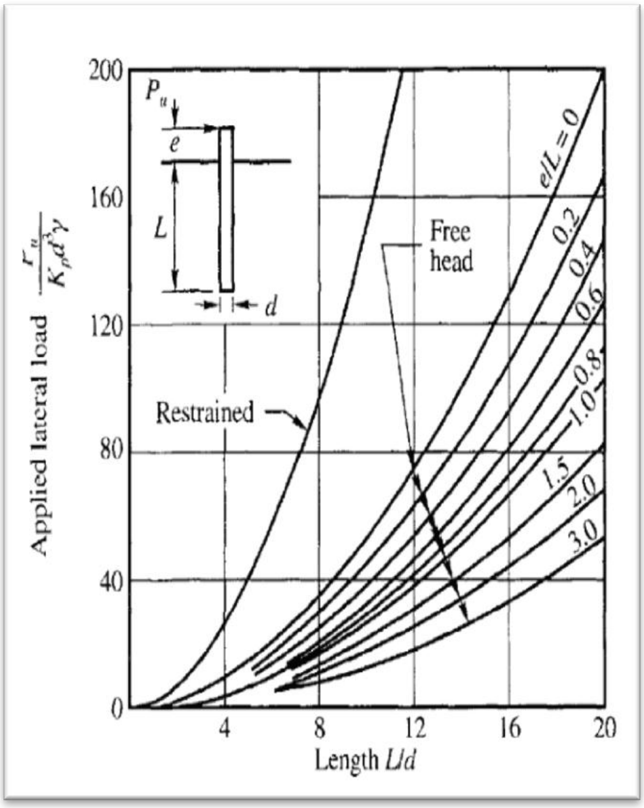

Fig. (4) for cohesive soil when pile is short. (After (Broms 1964))

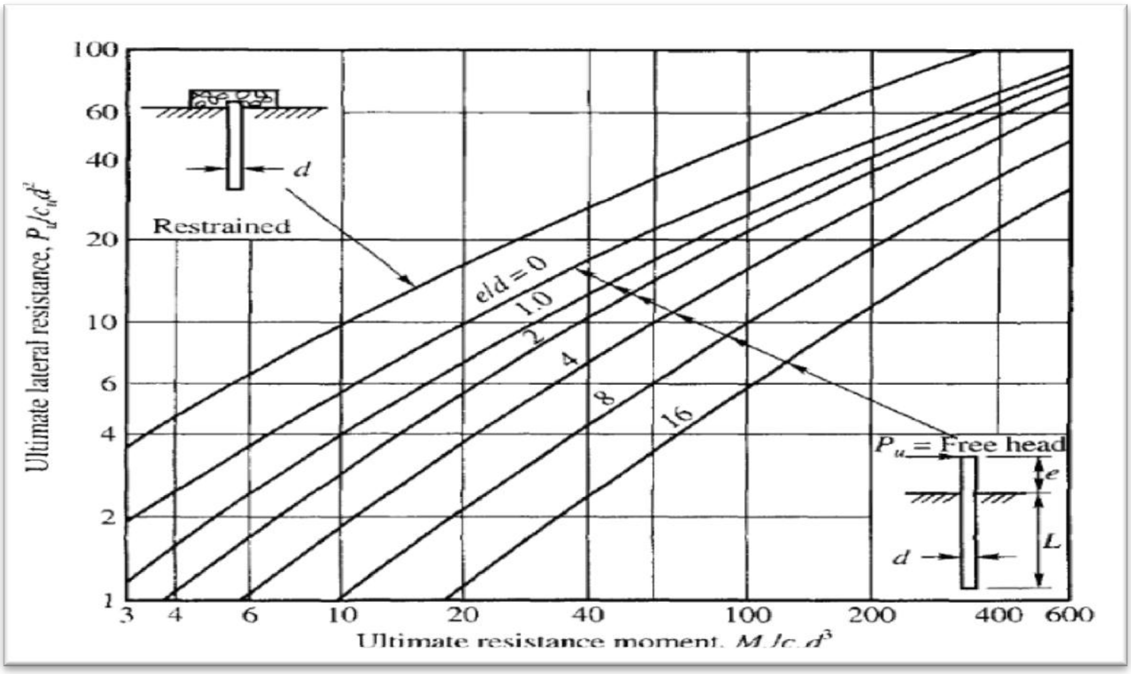

Fig. (5) for cohesive soil when pile is long. (After (Broms 1964))

5.GERMAN CODE DIN 4014(1990) [4]

Lateral load resistance of piles can be estimated by loading experiments or through knowledge obtained from previous loading tests performed under similar conditions. If piles are subjected to cyclic dynamic loading and/or alternating loads, this must be recreated as realistically possible in the tests, unless empirical values are provided. Loading should be continued until there is no longer a rise in strain. Creep under persistent loading should also be taken into account. The size and distribution of the Coefficient of Subgrade response must be calculated if the prescribed lateral displacement or

rotation of the pile head is not to be exceeded.

of lateral loading in the tests should be as near to the decision loads as practicable, with vertical loads being neglected. When piles are subjected to impact loads, the subgrade response coefficient, k, must be multiplied by three. When just a sufficient determination of the bending moment is required, the coefficients of subgrade are used.

The following equation can be used to calculate the response of the soil strata involved from **equation (15):**

K=E1/D (15)

Where:

K:is the coefficient of subgrade reaction.

E: is the modulus of stuffiness.

D: is the pile shaft diameter (not exceeding 1m a value of 1m also being assumed. Where D is actually greater).

is applicable to lateral displacements of up to 2 cm or 0.03 D, whichever is less.

Taking into consideration the size and sign of the wall friction angle, the stresses between the pile and the surrounding ground must not exceed the earth pressure at failure, Kp, as stated in DIN 4085. This calculation also assumes that the soil is neither temporarily or permanently removed, which might upset the balance between the piles, structure, and surrounding ground. NOTE: Failure under lateral loading is neglected since the loads that structures can handle involving a head displacement, y, or rotation, a, are considerably lesser than the load at failure.

6. JEAN-LOUIS BRIAUD METHOD [7]

The approaches are based on the observation given below.

Figure (8) depicts a hypothetical plot of soil resistance P per unit length of pile as a function of depth z. Because the P-z profile is sinusoidal (Baguelin et al. 1978; Briaud 1992) **[7]**, the soil resistance P alternates direction and basically balances itself out, except for a shallow zone near to the surface that contributes the most to the lateral resistance. **Figure (9)** shows the relation between L /lo and Dv .

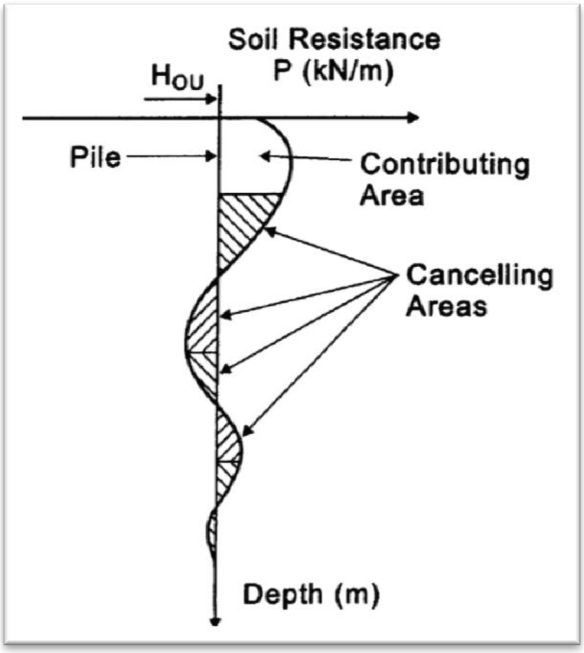

Fig.(8): Soli Resistance versus Depth Profile (After (Briaud Jean-Louis (1997))

Equation. (16) is based on the assumption that the pile is indefinitely long, whereas **Equation. (17)** assumes that the pile is stiff.

Comparison with the general solution (Hetenyi 1946) demonstrates that (l) is applicable if the pile length L is greater than 3Lo. Similarly, it can be demonstrated that (5) applies if the pile length L is less than Lo.

$$\mathbf{Dv} = \frac{\pi}{4}\mathbf{L_0} \qquad L \geq 3L_0 \qquad \mathbf{(16)}$$

$$\mathbf{Dv} = \frac{L}{3} \qquad L \leq L_0 \qquad \mathbf{(17)}$$

Where

L: is length of pile.

Dv: zero-shear depth.

Figure (9) shows the relation between L/lo and Dv .

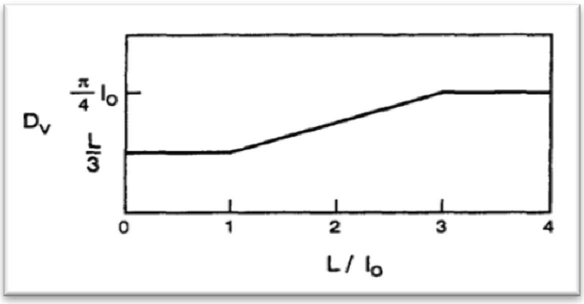

Fig.(9): Linear Interpolation for Zero-Shear Depth Dv. (After (Briaud Jean-Louis (1997))

Then taking **equation (18)**

$$\mathbf{Hou=Hou/3} \qquad \mathbf{(18)}$$

Where Hou : The lateral capacity is just used to show the method's dependability at tiny deflections. To find the deflection use **Figure (10)** by calculate the lateral capacity Hou.

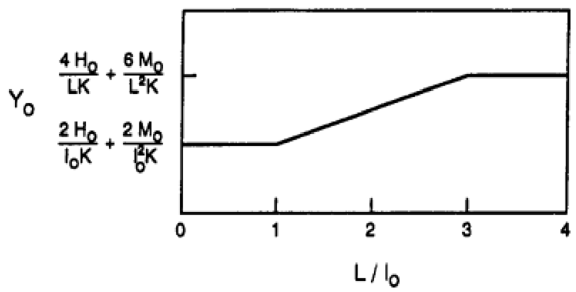

Fig.(10): Calculate the deflection (After (Briaud Jean-Louis (1997))

7. METHODOLOGY

In the current study, a database of 40 pile loading tests is employed. All of the case studies may be found
at Texas A&M 1977, Bagnolet, Brent Cross, Japan, Mustang Island, Cairo Monorails, Taiwan, Iraq, and Vakkayil.. As illustrated in **Table (5), and Figure (11),** the length of the pile ranges from 0.5 m to 36.6 m, while the width /diameter (D) ranges from 0.273 m to 1.8 m. **Table (6)** summarizes the data from those case studies, including the soil type, length, diameter, and projected.

Table (5) the rages of length and diameter for case studies.

case	D (m)	L (m)
Edmonton/U4	0.324	24
Edmonton/C1	0.324	24
Edmonton/C2	0.324	24
Edmonton/C3	0.324	24
New Orleans/TPU	0.356	21
New Orleans/CP1	0.356	21
New Orleans/SP3	0.324	21
Plancoet	0.28	6.1
Baytown	0.61	11.9
Sabine	0.324	11
Lake Austin	0.324	12.2
Texas A&M 1977	0.915	6.1
Texas A&M 1978	0.915	4.6
U. of Houston	0.273	11.8
Baytown/pile 2	0.61	36.6
Baytown/pile 3	0.51	29.6
Lock & Dam 26 (83)	0.356	20.4
Lock & Dam 26 (83)	0.356	20.4
Lock & Dam 26 (78)	0.356	15.2
Lock & Dam 26 (78)	0.356	15.2
Bagnolet	0.43	3.35
Bagnolet	0.43	5.05
Bagnolet	0.43	6.1
Houston	0.762	12.8
Brent Cross	0.406	16.5
Japan	0.305	5.18
Mustang Island	0.61	21
Cairo Monorails	1.8	19.6
Baghdad	0.016	0.5
Baghdad	0.016	0.5
Kurichikkal bridge	1.2	7.37
parallel bridge to pullut 1	1.2	7.37
Vakkayil bridge	1.2	10.62
parallel bridge to pullut 2	1.2	10.62
Ezhavapalam bridge	1.2	8.46
Chengalayi bridge	1.2	8.81
Taiwan	1.5	34.9
iraq	0.6	12
iraq	0.6	15
iraq	0.6	18

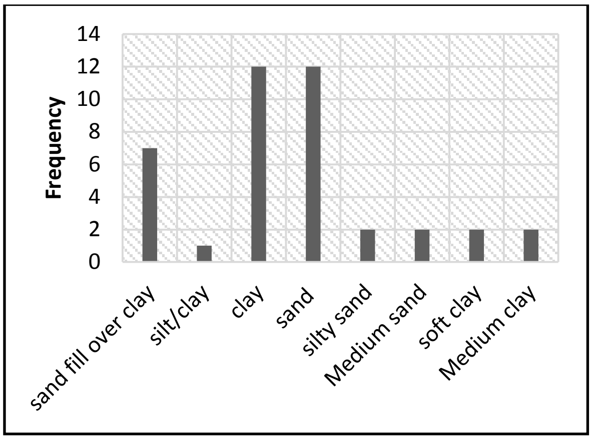

Fig.(11): Case studies according to type of soil

The allowed lateral load (0.02D) and ultimate horizontal load (0.1D) were computed using numerous international codes. From the normalization, we can determine which of the codes and empirical equations produces the most accurate results. **Table (6)** displays the findings of the allowed and ultimate lateral loads.

Table (6) The lateral loads on piles , soil type and each study reference.

NO. CASE	Egyptian code	indian code		AASHTO,British Standard , and Candian code,			Jean-Louis Briaud method
CASE	case	soil type	Ep Ip	(m)	(m)	(KN)	Reference
1	**Edmonton/U4**	sand fill over clay	1.87E+04	0.324	24	90	[7]
2	**Edmonton/C1**	sand fill over clay	1.87E+04	0.324	24	97	[7]
3	**Edmonton/C2**	sand fill over clay	1.87E+04	0.324	24	93	[7]
4	**Edmonton/C3**	sand fill over clay	1.87E+04	0.324	24	93	[7]
5	**New Orleans/TPU**	sand fill over clay	1.10E+04	0.356	21	104	[7]
6	**New Orleans/CP1**	sand fill over clay	2.81E+04	0.356	21	137	[7]
7	**New Orleans/SP3**	sand fill over clay	2.44E+04	0.324	21	145	[7]
8	**Plancoet**	silt/clay	2.74E+04	0.28	6.1	38	[7]
9	**Baytown**	clay	1.43E+05	0.61	11.9	260	[7]
10	**Sabine**	clay	3.68E+04	0.324	11	55	[7]
11	**Lake Austin**	clay	3.50E+04	0.324	12.2	77	[7]
12	**Texas A&M 1977**	clay	7.22E+05	0.915	6.1	756	[7]
13	**Texas A&M 1978**	clay	7.22E+05	0.915	4.6	556	[7]
14	**U. of Houston**	clay	1.34E+04	0.273	11.8	73	[7]
15	**Baytown/pile 2**	sand	2.60E+05	0.61	36.6	712	[7]
16	**Baytown/pile 3**	sand	4.59E+04	0.51	29.6	422	[7]
17	**Lock & Dam 26 (83)**	sand	6.10E+04	0.356	20.4	245	[7]
18	**Lock & Dam 26 (83)**	sand	6.10E+04	0.356	20.4	272	[7]
19	**Lock & Dam 26 (78)**	sand	6.10E+04	0.356	15.2	258	[7]
20	**Lock & Dam 26 (78)**	sand	3.10E+04	0.356	15.2	192	[7]
21	**Bagnolet**	clay	25500	0.43	3.35	80	[8]
22	**Bagnolet**	clay	25500	0.43	5.05	80	[8]
23	**Bagnolet**	clay	25500	0.43	6.1	80	[8]
24	**Houston**	clay	400000	0.762	12.8	450	[8]
25	**Brent Cross**	clay	51400	0.406	16.5	100	[8]
26	**Japan**	clay	6868	0.305	5.18	15	[8]
27	**Mustang Island**	sand	163000.0	0.6	21.0	21.0	[8]
28	**Cairo Monorails**	silty sand	25000	1.8	19.6	1700	[9]
29	**Baghdad**	sand	0.215	0.016	0.5	0.08	[10]
30	**Baghdad**	sand	0.215	0.016	0.5	0.12	[10]
31	**Kurichikkal bridge**	Medium sand	3218.81	1.2	7.37	219.46	[11]
32	**parallel bridge to pullut 1**	Medium sand	3218.81	1.2	7.37	233.63	[11]
33	**Vakkayil bridge**	soft clay	3218.81	1.2	10.62	144.5	[11]
34	**parallel bridge to pullut 2**	soft clay	3218.81	1.2	10.62	191.05	[11]
35	**Ezhavapalam bridge**	Medium clay	3218.81	1.2	8.46	218.4	[11]
36	**Chengalayi bridge**	Medium clay	3218.81	1.2	8.81	184.7	[11]
37	**Taiwan**	silty sand	6958136.8	1.5	34.9	3000	[12]
38	**iraq**	sand	165404.8	0.6	12	210	[13]
39	**iraq**	sand	165404.8	0.6	15	245	[13]
40	**iraq**	sand	165404.8	0.6	18	270	[13]

	Hult	Hall	Hult	Hall	Hult	Hall	Hult	Hall
1	104.51	104.51	104.51	19.39	99.45	19.89	59.00	90.00
2	104.51	104.51	104.51	19.39	99.45	19.89	59.00	97.00
3	104.51	104.51	104.51	19.39	99.45	19.89	59.00	93.00
4	104.51	104.51	104.51	19.39	99.45	19.89	59.00	93.00
5	174.48	174.48	174.48	21.34	103.22	20.64	140.00	104.00
6	135.15	135.15	135.15	27.17	132.87	26.57	177.00	137.00
7	155.72	155.72	155.72	29.80	144.92	28.98	157.00	145.00
8	35.41	35.41	35.41	6.17	47.82	9.56	54.00	38.00
9	245.32	245.32	245.32	53.98	442.80	88.56	299.00	260.00
10	46.10	46.10	46.10	10.79	115.47	23.09	54.00	55.00
11	71.39	71.39	71.39	15.83	128.07	25.61	84.00	77.00
12	1064.74	1064.74	1064.74	152.83	715.04	150.21	792.00	756.00
13	1064.74	1064.74	1064.74	152.83	558.15	111.63	586.00	556.00
14	70.42	70.42	70.42	15.67	87.94	17.59	67.00	73.00
15	755.38	755.38	755.38	143.13	709.12	141.82	762.00	712.00
16	436.11	436.11	436.11	85.95	418.34	83.67	399.00	422.00
17	246.85	246.85	246.85	51.82	241.97	48.39	217.00	245.00
18	246.85	246.85	246.85	51.82	241.97	48.39	217.00	272.00
19	246.85	246.85	246.85	51.82	250.52	50.10	312.00	258.00
20	204.00	204.00	204.00	39.53	194.72	38.94	263.00	192.00
21	131.37	131.37	131.37	17.24	79.41	15.88	72.03	80.00
22	131.37	131.37	131.37	17.24	93.37	18.67	54.29	80.00
23	131.37	131.37	131.37	17.24	112.79	22.56	98.36	80.00
24	462.42	462.42	462.42	88.91	743.22	148.64	420.04	450.00
25	108.44	108.44	108.44	19.05	271.98	54.40	133.99	100.00
26	20.22	20.22	20.22	4.75	48.19	9.64	24.34	15.00
27	460.15	460.15	460.15	51.35	241.14	48.23	210.57	210.57
28	1746.02	1746.02	1746.02	337.50	1711.80	342.36	1442.91	1700.00
29	0.14	0.14	0.14	0.01	0.09	0.02	0.20	0.08
30	0.14	0.14	0.14	0.01	0.09	0.02	0.20	0.12
31	288.57	288.57	288.57	47.10	223.83	44.77	221.10	219.46
32	288.57	288.57	288.57	47.10	223.83	44.77	221.10	233.64
33	288.57	288.57	288.57	47.10	225.77	45.15	318.60	144.51
34	288.57	288.57	288.57	47.10	225.77	45.15	318.60	191.05
35	288.57	288.57	288.57	47.10	1218.24	243.65	253.80	218.49
36	288.57	288.57	288.57	47.10	1268.64	253.73	264.30	184.74
37	2853.31	2853.31	2853.31	607.81	3047.33	609.47	3363.90	3000.00
38	255.76	255.76	255.76	47.64	432.00	68.40	211.34	210.00
39	255.76	255.76	255.76	47.64	540.00	108.00	211.34	245.00
40	255.76	255.76	255.76	47.64	648.00	129.60	211.34	270.00

Table (7) The results of allowable lateral load and ultimate lateral load

8. Discussion of the results

In cases number (12,13) the soil was clay soil Egyptian code gives a far results from the horizontal load measurements the piles were a rigid pile .Also, in case number 37 in Taiwan the pile is elastic pile in silty sand soil results of normalization is far from the field measurements. The greater in the horizontal load , the further the results from the test measurements in silty sand soil .

In summary, from the 40 cases the case in which the soil was clay and silty sand soil when calculated the horizontal load using Egyptian code gives the results was far from the measurements. Also, from chart can observed that when calculated the horizontal load in sand ,silty clay, and sand fill over clay the Egyptian code give seemly equal to the measurements from field and laboratory test.

We can find that the dispersion for all cases when using a normalized values reflected in coefficient of determination value (R^2 =0.9682) it almost a close results to the test measurements according to the 40 cases. **Figure (12)** conclude the results.

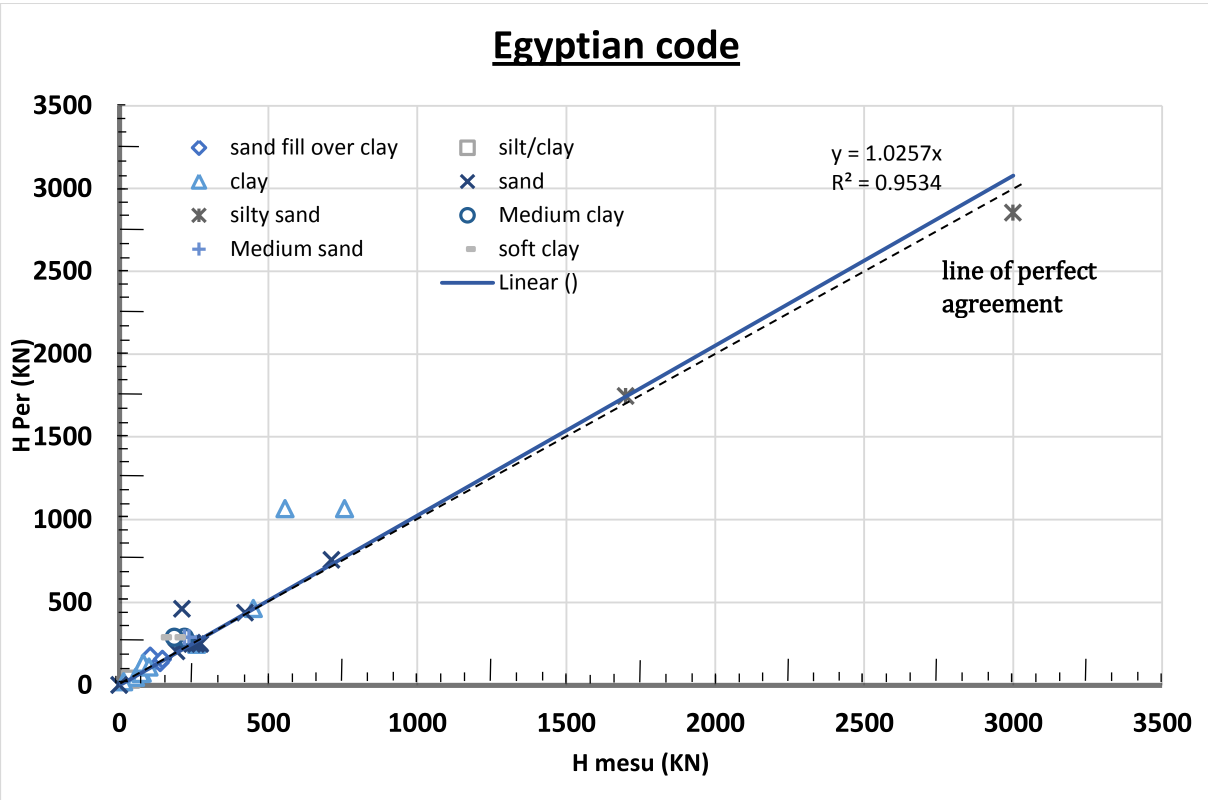

Fig. (12) Results using Egyptian code

When use Indian code to calculate the lateral load in 40 cases it almost gives the same test measurements values. Maybe because Indian code empirical equations use a modifier factor (depth of fixity). Except only case number 12 gives a high value of lateral load than measurements test.

In summary, the Indian code gives fairly accurate results in the 40 cases when the soil is sandy and clay soils. The dispersion for all cases when using a normalized values reflected in coefficient of determination value (R^2 =0.9962) it almost a closest results to the test measurements according to the 40 cases . **Figure (13)** summarize the results.

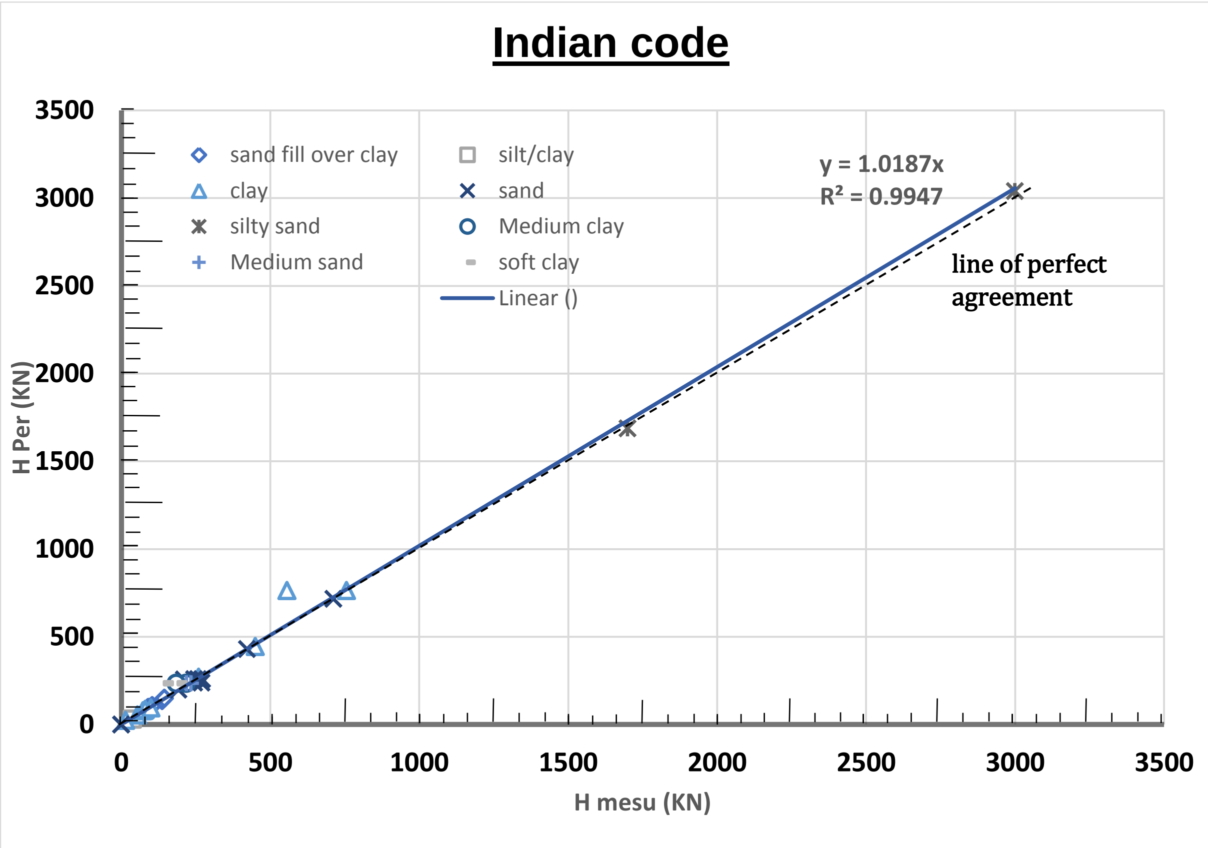

Fig. (13) Results using Indian code

Canadian code, British Standard (2002),and AASHTO (2005) they are use Brom's method to calculate the lateral load .In case number 28,and 37 the pile was long pile and the soil is silty sand when use Canadian code, British Standard (2002),and AASHTO (2005) to calculate the lateral load they gives a low values than the values of lateral load of measurements test. Also from the 40 cases when use Canadian code, British Standard (2002),and AASHTO (2005) to calculate horizontal load in clay soil it gives a variable values regarding the measurements from field and laboratory test. Case 35, and 36 the medium clay soil when use empirical equation gives a high values than measurements.

In summary, the Canadian code, British Standard (2002),and AASHTO (2005) codes when used the in the 40 studies gives fairly accurate results in the sandy Soils and soft clay rather than clay ones. The dispersion for all cases when using a normalized values reflected in coefficient of determination value (R^2 =0.866) it almost a close results to the test measurements according to the 40 cases. In **figure (14)** we can see from the chart the results using Canadian code, British Standard (2002),and AASHTO (2005) codes.

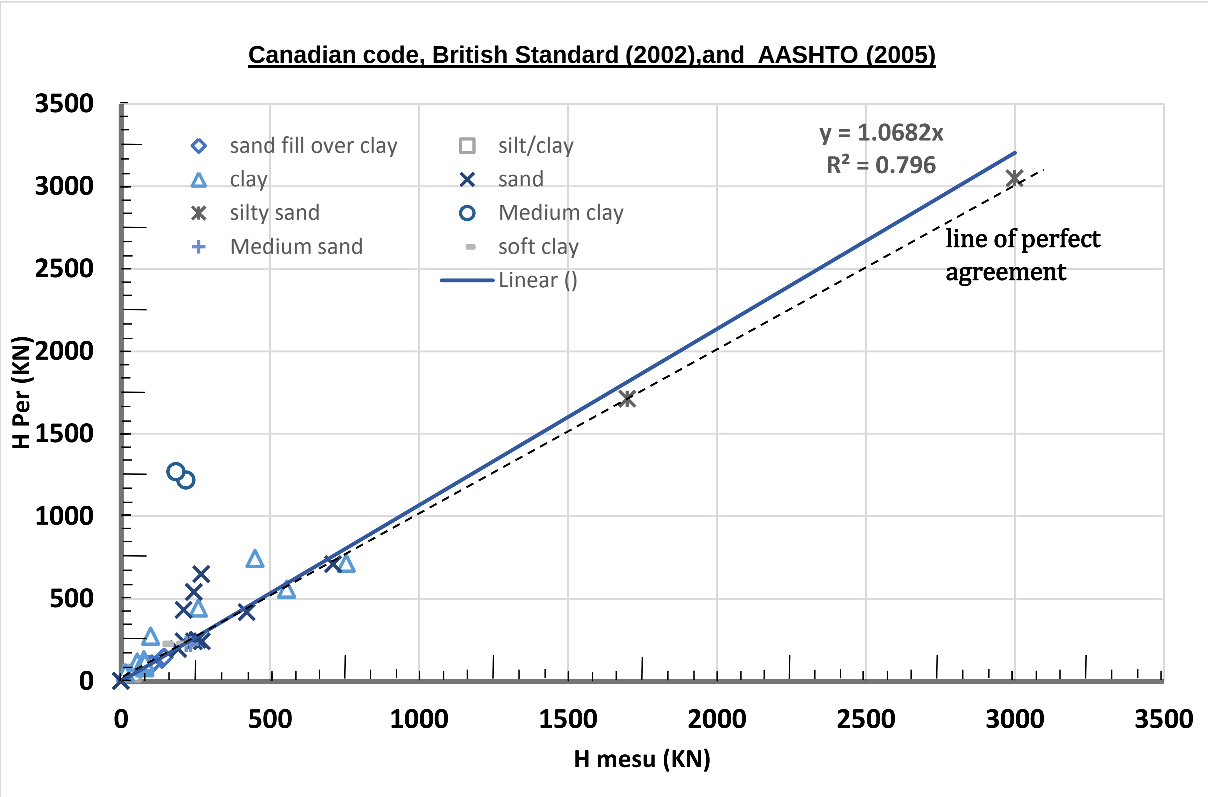

Fig. (14) Results Canadian code, British Standard (2002),and AASHTO (2005).

In case number 28, and 37 when use the Jean-Louis-Briaud method in silty sand when the pile is long they give a variable values results than measurements values. Also in case numbers 33, and 34 the soil was soft clay when use empirical equations in Jean-Louis-Briaud method the values of lateral load is higher than measurements values.

In summary, Jean-Louis-Briaud method equations in the 40 cases give fairly accurate results in the sandy Soils and soft clay rather than silty sand ones. The dispersion for all cases when using a normalized values reflected in coefficient of determination value (R^2 =0.985) it almost a close results to the test measurements according to the 40 cases. In figure 13 we can see from the chart the results using Jean-Louis-Briaud method.

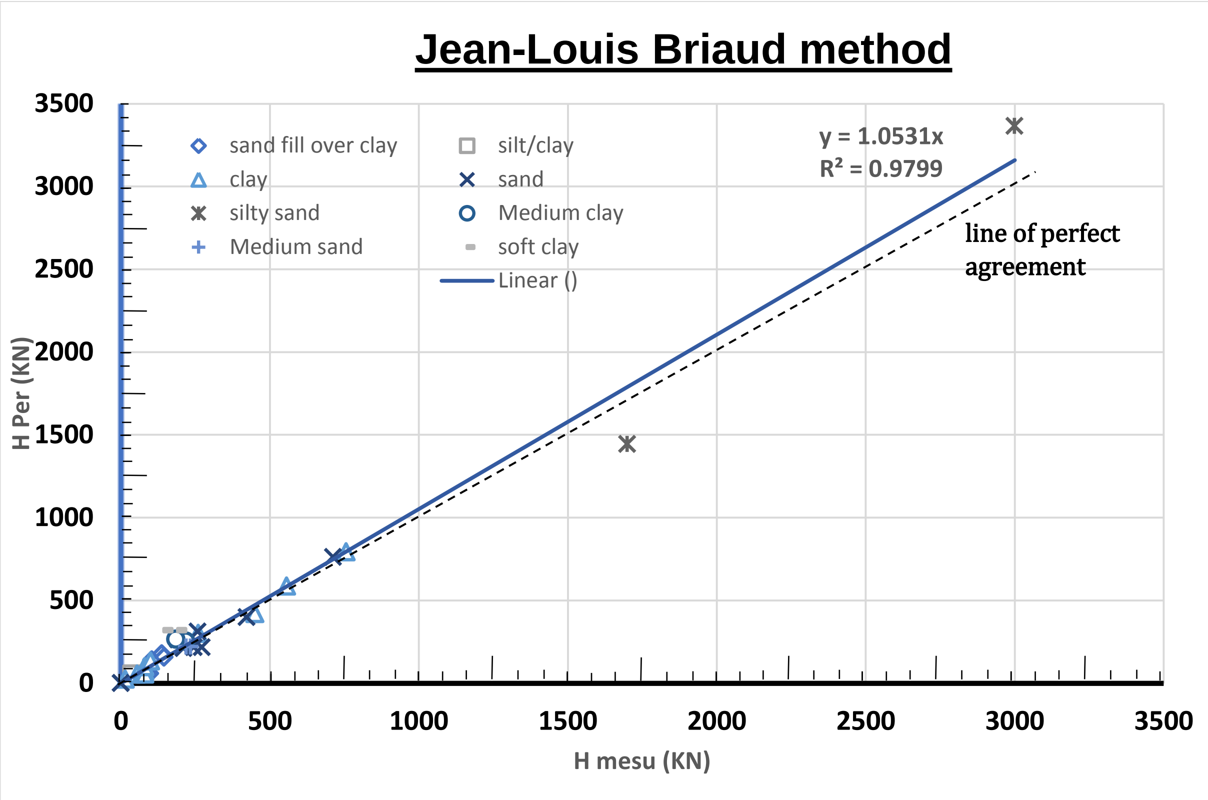

Fig. (15) Results using Jean-Louis Briaud method

9.Concluding remarks

Based on the analysis performed in this paper for the 40 cases study cases for laterally loaded piles the following remarks could be drawn:

1- All code results for the case studies under investigation were found that Indian code results are closest to the field results, followed by the Indian code, then the Jean-Louis Briaud method , Egyptian code finally AASHTO, British Standard, and Canadian code .

2- The Egyptian code and canadian code, British Standard (2002),and AASHTO (2005) gives in case number 12 and 13 in clay soil variable values than the field measurements values. Also in cases number 28, and 37 in silty sand soil the Egyptian code, Canadian code, British Standard (2002),and AASHTO (2005), and Jean-Louis Briaud method gives far values for lateral load than test measurements values.

3- Jean-Louis Briaud method , Egyptian code , and Indian code in the 40 cases when the soil is clay the values seems to be equal to field measurements values.

4- Indian code in the 40 cases gives the closest results maybe because the empirical equations uses a modifier factor for the depth of fixity effect.

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